Attachment 18

Proposed Bay d'Espoir Hydro Generating Unit 8 – Hydraulic Analysis of the Conveyance

SNC-Lavalin Inc.

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NEWFOUNDLAND AND LABRADOR HYDRO

Proposed Bay d'Espoir Hydro Generating Unit 8

HYDRAULIC ANALYSIS OF THE CONVEYANCE SYSTEM

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1 INTRODUCTION

1.1 Background

SNC-Lavalin inc (SLI) was retained by Newfoundland and Labrador Hydro (NLH) to complete an hydraulic analysis of the water conveyance (RFP Scope Item 1) together with a class 3 cost estimate and project execution schedule (RFP Scope Item 2) for the Bay D'Espoir Hydro Generating Unit 8 (RFP 2017-70845 JW).

This is the engineering report in accordance with Scope Item 1.

1.2 Description of existing facilities

The existing development at Bay D'Espoir includes a reservoir, a spillway and two Powerhouses.

Powerhouse 1 has six generating units of 75 MW nominal capacity each and three individual intakes and penstocks each supplying two units through a bifurcation near the powerhouse. The first four units were commissioned in 1967 (phase 1) and the last two units (phase 2) were commissioned in 1977. A single headrace canal provides water to the three intakes and the powerhouse discharges via a 4.5 km long tailrace channel which flows into Fortune Bay.

Powerhouse 2 includes a single unit of 150 MW nominal capacity. Water is provided by a separate headrace canal, intake and penstock. This powerhouse discharges in its own tailrace channel connecting Powerhouse 2 to the tailrace channel of Powerhouse 1. This powerhouse was commissioned in 1977 (phase 3) and was constructed for the future installation of a second 150 MW unit. In this regard, rock excavation for the second unit was completed and the downstream portion of the draft tube, with the draft tube gates guides, was constructed to prevent disruption to the operation of the existing Unit 7 while constructing Unit 8.

The reservoir dams were later raised to increase the maximum operating water level by 2.0 m and flood maximum water level was set 3.5 m higher than the original maximum water level.

The headrace canal, the intake, the penstock and the downstream portion of the tailrace channel of Powerhouse 2 were, however, designed and built to only accommodate existing Unit 7. At that time, it was planned that the headrace and tailrace channels would be enlarged when building Unit 8 and that a new intake and penstock would be required.

NLH is now considering the option of adding an additional unit (Unit 8) to Powerhouse 2.

1.3 Objectives

The present report presents the analysis of the hydraulic conveyance system to determine the feasibility and the economics of the following alternatives:

- Install a bifurcation with insulation valves and use the existing intake and penstock to supply water to both Units 7 and 8.

- Add a dedicated intake and penstock to supply Unit 8.

The report also discuss the modifications required to the existing headrace and tailrace channels in order to efficiently transit the increased flow. Finally, recommendations regarding the hydraulic conveyance system for Unit 8 are presented.

1.4 Exclusions

The analysis did not consider the effect of the additional flow on the common tailrace from the confluence of the two tailraces to the bay. The probable cost to complete this task is estimated at 10 000$.

2 HEADRACE CANAL

2.1 Reservoir water level

The reservoir was initially built to operate between elevation 178.3 m (585') and 180.75 m (593'). Later, the maximum operating level was raised to 182.7 m (599.5') and the extreme flood level to 184.2 m (604.33') while maintaining the same minimum water level. The following reservoir water levels were hence used for the present study:

Minimum Water Level:	178.3 m (585')
Maximum Spring Water Level:	182.7 m (599.5')
Maximum Fall & Winter Level:	180.25 m to 182.25 m (591.5' to 598')
Max. Flood Level:	184.2 m (604.33')

The maximum water level during winter of 182.25 m is reached with no snow cover and reduces linearly to 180.25 m when the snow cover reaches an equivalent of 230 mm of water.

2.2 Existing headrace canal geometry

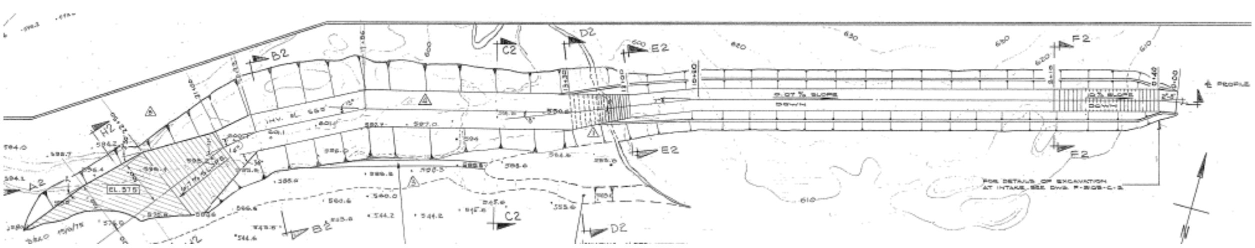
The existing channel is divided into three (3) sections, a funnelling entrance, an upstream portion excavated in overburden and a downstream portion excavated in rock.

The invert of the funnelling entrance has been set at elevation 175.25 m (575') with a width of 61 m (200') upstream and slopes down to elevation 172.2 m (565') with a width of 24.4 m (80') at its downstream end. The canal sides have been excavated at a 2H:1V slope.

The upstream portion, 274 m (900') long, has its invert set at 172.2 m (565') with a width of 24.4 m (80') and side slopes of 2H:1V. It includes a transition 39.6 m (130') long at its downstream end where the invert slopes down to elevation 171.6 m (563') and the width reduces from 24.4 m (80') to 15.25 m (50').

The downstream portion is 366 m (1,200') long and is excavated in rock with an invert 15.25 m (50') wide at elevation 171.6 m (563'). The canal side walls are excavated with a 1H:6V slope. Near the intake, the invert gradually dives to elevation 163.4 m (536') and enlarges to 15.85 m while the walls become vertical.

Figure 2.1 – Existing canal geometry



2.3 Flow velocities in existing headrace canal

In open water conditions (and no ice cover), velocities up to 3 to 3.5 m/s would be acceptable and would result in additional head losses which, considering the length of the existing headrace canal, would be less than 0.5 m.

In winter, depending on the operation patterns of the power plant, the reservoir level and the ambient temperature, the ice cover versus flow velocities may introduce very limiting operating constraints.

If the flow velocities are consistently maintained above 1.0 m/s (base production instead of peak production), then, by thermal erosion, no ice cover would form in this portion of the channel and ice cover would not be a problem. Considering the length of the channel and the fact that an ice cover forms on the reservoir, no frazil ice formation would be anticipated.

If the flow velocities are consistently maintained below 0.65 m/s, a stable ice cover would form and resist thermal erosion. So, except for the periods of ice cover formation and thawing, no constraints related to the ice cover would be anticipated whatever the operating pattern. During ice cover formation, however, velocities lower than 0,65 m/s may be temporarily required until the ice cover has reach a sufficient thickness (few inches) to resist against ice brake up.

Velocities ranging from 0.65 m/s to 1.0 m/s can be acceptable for short periods (few hours a day depending on the ambient temperature, longer periods being possible when the temperature is colder) once the ice cover is sufficiently resistant if the period at peak flow is short enough so not to substantially thermally erode the ice cover and if the period at low flow is long enough to allow for rebuilding the ice cover.

Variable flow velocities with peaks above 1.0 m/s present the worst condition since there is a high probability that the ice cover will breakup and accumulate at the intake and may partially block the trash racks by being sucked down, especially at low water level operations since the setting of the intake is marginal according to Gordon submergence criteria (see chapter 3).

For this study, velocities were estimated up to the funnelling entrance of Unit 7 & 8 headrace canal. Upstream this point, the flow of Units 7 & 8 joins with the flow of Units 1 to 6 and a 2D model as well as bathymetry of the reservoir is required to estimate the flow velocities and patterns. Such study was out of the present scope but would, however, be required in a future phase to assess that there is no impact to the ice cover in the reservoir by adding Unit 8 and thus increasing the maximum overall flow from approximately 400 m^3/s to 500 m^3/s. If velocities were found to be too high for a stable ice cover upstream of the channels, mitigation measures such as ice booms or additional excavations could be implemented depending on the velocities.

Assuming a flow of approximately 100 m^3/s for Unit 7 (which corresponds to 150 MW power generation) and 200 m^3/s for combined operation of Units 7 & 8, velocities were estimated in the upstream portion (excavated in overburden) and downstream portion (excavated in rock) of the channel.

For Unit 7 (alone), the maximum velocities are ranging in the upstream portion of the channel from 0.22 m/s (winter maximum water level) to 0.43 m/s (minimum water level) and from 0.55 m/s (maximum winter water level) to 0.87 m/s (minimum water level) in the downstream portion.

With the addition of Unit 8 and with the headrace canal maintained to its present dimensions, maximum velocities (Units 7 & 8 at max. output) would increase to 0.44 m/s (maximum winter water level) and 0.86 m/s (minimum water level) in the upstream portion of the canal and to 1.1 m/s (maximum winter water level) and 1.75 m/s (minimum water level) in the downstream portion of the canal.

The current flow velocities in the headrace canal for Unit 7 alone are acceptable with the present conditions but the dimensions of the channel are near acceptable limits especially if the reservoir is often drawdown to its minimum operating level and if the unit runs at its maximum output of 150 MW for several hours a day for peaking production.

It is most probable that maintaining the present channel as is might introduce operating constraints during the winter season that may negate all the benefits of adding Unit 8. In this regard, without knowing the future pattern of operation for Units 7 & 8 and neither knowing the future reservoir level management strategy, it is recommended at this point to enlarge the headrace canal on its entire length to eliminate all negative operating constraints.

Detailed optimization of the enlargement required regarding the future reservoir level management as well as the future Unit 7 & 8 operating pattern (base production versus peaking production) is recommended in a future phase. This optimization could, in some cases, yield to a substantial reduction in the required excavation and even eliminate the need for additional excavation in some zones of the channel, especially if it is found that the reservoir elevation never reaches the minimum water level in winter time (ice cover period).

2.4 Enlargement of the headrace canal

An enlargement of the headrace canal was considered in this analysis to maintain velocities under 0.65 m/s for full output at the minimum water level in the upstream portion of the canal excavated in overburden and under 0.85 m/s (actual maximum velocity) in the downstream portion excavated in rock.

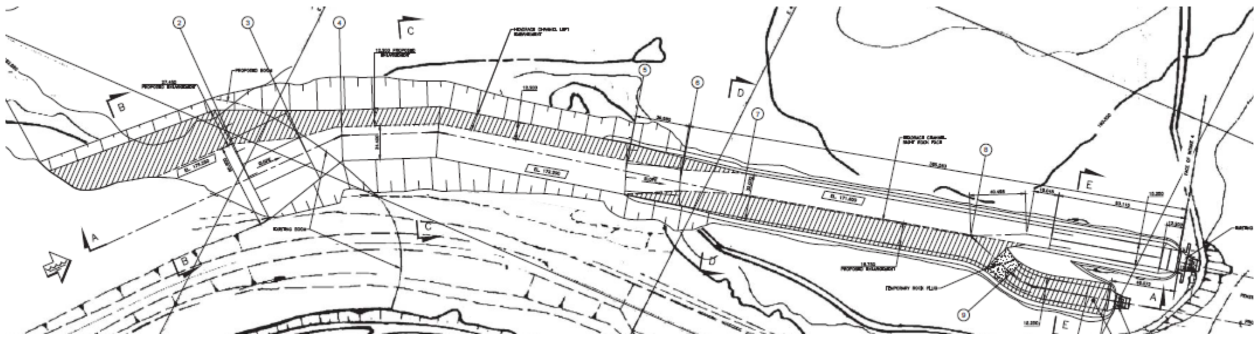
In this regard, the upstream portion was enlarged on the north side by 12.2 m (40') and the downstream portion by 16.75 m (55') on the south side which as a result, straightens the alignment of the original canal.

A bifurcation on the south side of the canal was also introduced in the headrace canal approximately 125 m upstream of the existing intake of Unit 7. From this point, the headrace canal is divided into two approach canals: the existing one for Unit 7 and a new approach canal for Unit 8. Maintaining a temporary rock plug at the upstream of the new Unit 8 approach canal will allow for the excavation of most of Unit 8 approach canal and intake under dry conditions and prevent disruptions to the operation of Unit 7.

Once the new intake have been commissioned, the approach canal between the intake and the rock plug would be flooded then he rock plug would be excavated under wet conditions.

The width and depth of the new intake approach canal for unit 8 were considered identical to the existing Unit 7 channel. The new channel has been implemented 36.5 m south of the existing channel to maintain a rock pillar width of 21.3 m between the two canals. During construction, this pillar and the temporary rock plug will be maintaining the reservoir closure integrity. The new intake approach channel ends at the new intake approximately 45 m upstream of the existing intake for Unit 7. This implementation was necessary to ensure the closure of the reservoir within the rock walls of the new channel since the rock surface slopes down further downstream. The reservoir closure would have otherwise required the construction of a concrete wall on the right side of the channel.

Figure 2.2 – Proposed canal layout



3 INTAKE

3.1 Unit 7 existing intake supplying both Units

Settings of Unit 7 intake were originally determined for a flow of 102 m^3/s corresponding to the maximum 150 MW output and minimum water level. These settings were also compared to Gordon's symmetrical intake setting criteria.

At the trash racks:

- Gross area at trash racks: 154.1 m^2
- Gross velocity at trash racks: 0.65 m/s
- Trash rack lintel elevation: 177.4 m
- Gordon symmetrical required submersion: 1,3 m
- Existing submersion (at minimum water level): 0,9 m

At the gate:

- Flow area at the gate: 26.85 m^2
- Velocity at intake gate: 3.8 m/s
- Intake gate lintel elevation: 170.1 m
- Gordon symmetrical required submersion: 4,7 m
- Existing submersion (at minimum water level): 8.2 m

Based on these results, it is possible that some intermittent vortex could be observed at the minimum water level resulting in floating free pieces of ice and debris being sucked down on the trash racks. However, because of the submersion at the gate, no air is expected to be sucked in the penstock and affect the flow in the machine.

If the existing Unit 7 intake was to be used for both units then the flow in the intake would rise to 200 m^3/s for a maximum output of 300 MW and the settings would compare to the Gordon's criterion for symmetrical flow as follows:

At the trash racks:

- Gross velocity at trash rack: 1.3 m/s
- Gordon symmetrical required submersion: 2.6 m
- Existing submersion (at minimum water level): 0,9 m

At the gate:

- Velocity at intake gate: 7.6 m/s
- Gordon symmetrical required submersion: 9.4 m
- Existing submersion (at minimum water level): 8.2 m

It is thus expected that a permanent vortex would be present at low water level. Under these conditions there is a possibility that air could be sucked into the penstock and make is way down to the generating unit adversely affecting its efficiency and behaviour. Also, floating free pieces of ice and debris would be sucked down on the trash racks, creating obstruction and additional head losses at the intake that may require to operate at lower capacities until the ice in the trash racks as melt down.

In this regard, a shared intake would significantly limit the simultaneous operation of both Units 7 and 8.

3.2 Dedicated Unit 8 intake

The four existing intakes for Units 1 to 7 are nearly identical and use the same stoplogs for the maintenance for the gates. Minor changes were however introduced to Unit 8 intake to adapt to the existing topography and to the new reservoir water levels:

- The intake deck was raised to elevation 184.4 m (605') to account for the reservoir raise that was introduced years ago.
- The intake was implemented about 45 m upstream of the current location of Unit 7 intake to ensure that the reservoir closure is maintained within the rock walls of the new

entrance channel. South of the existing intake, within the axis of the new intake, the rock is at a lower elevation and, therefore, a concrete wall would have been required on the south side of the channel to ensure the reservoir closure had the intake been implemented at the same location as Unit 7 intake.

- The length of penstock embedded in concrete was increased by 45 m downstream of the intake. This was considered to bring the end of the penstock embedment at the same location as for Unit 7, thus allowing building a deep fill above the penstock on a longer distance to maintain the existing access road at elevation 184,4 m to Unit 7 intake.

With the planned enlargement of the head race channel and the addition of the new approach channel for the Unit 8 intake, the incoming flow at both Intakes 7 and 8 would be asymmetrical. The setting of both intakes had thus to be rechecked for Gordon asymmetrical flow criteria. The results are shown hereafter for a separate intake:

At the trash racks:

- Gross velocity at trash racks: 0.65 m/s
- Gordon asymmetrical required submersion: 1,7 m
- Existing submersion (at minimum water level): 0,9 m

At the gate:

- Velocity at intake gate: 3.8 m/s
- Gordon asymmetrical required submersion: 6,3 m
- Existing submersion (at minimum water level): 8.2 m

The submergence of the gate is adequate and thus no air should be sucked into the penstock. However, the submergence of the trash racks may be slightly more problematic at minimum reservoir water level as floating ice would be more prone to be sucked down on the trash racks.

In this regard, it is recommended to set the trash racks of new Unit 8 intake deeper by 1,0 m and to enlarge the headrace canal. This will provide a stable ice cover for all operating conditions and thus prevent floating ice to get sucked down into the trash racks.

It is not possible to optimize the intake location without a precise topography of the site and a good knowledge of the surrounding rock topography, nor possible to be sure that the reservoir closure within the new rock channel is guaranteed and no side concrete wall is required.

It is recommended that geotechnical investigations and LIDAR imagery of the site be completed in the next phase of the project to facilitate the detailed design phase of the project.

4 PENSTOCK

4.1 Unit 7 existing penstock

The existing penstock has a total length of 1,020 m (3,346') from the intake to the powerhouse. The inside diameter varies from 5.18 m (17') at the intake to 3.76 m (12' 4'') at the powerhouse with intermediary sections of 4.72 m (15' 6'') and 4.42 m (14' 6'') respectively.

The penstock was originally designed for a flow of 102 m^3/s, with internal flow velocities ranging from 4.85 m/s at the intake to 9.2 m/s at the powerhouse.

The penstock was also designed for a normal water hammer overpressure of 30% of the maximum original gross head of 179.5 m (589') for a total pressure of 234.5 m at the powerhouse (stresses limited to minimum of 0.60 fy and 0.38 fu). This loading case corresponds to the wicket gate closing time of Unit 7 set to 25 seconds combined with the operation of the pressure relief valve added to the hydraulic circuit and having a capacity of 10.2 m^3/s.

In addition to this normal loading case, an extreme water hammer loading case corresponding to a total pressure of 362.5 m (1190') at the powerhouse was also used to account for the extreme case of a failure of the governor system and a much faster closure of the wicket gates without the pressure release valve. For this case however, much higher stresses were accepted (minimum of 0.96 fy and 0.61 fu).

Head losses and water hammer overpressure have been estimated for the existing penstock using the new maximum reservoir flood level of 184.2 m, corresponding to a gross head of 183.0 m (600' 6''). A wicket gate closing time of 24.5 seconds, combined with a cushioning time of 25 seconds for a total closure time of 47 seconds, as described in the existing governor operation and maintenance manual were assumed.

Head losses were estimated to 5.65 m and the total water hammer maximum pressure was found to be below the 233.5 m design normal water hammer head (30% overpressure).

4.2 Penstock alternatives

4.2.1 Bifurcation located upstream of powerhouse

Flow velocities, head losses and water hammer overpressure were assessed for the alternative of installing a bifurcation immediately upstream of the powerhouse.

Flow rates with both units at maximum output (102 m^3/s for each unit) were assumed for this case. It was found that velocities ranged from 9.7 m/s at the intake to 13.3 m/s upstream of the bifurcation, while remaining around 9.2 m/s downstream of the bifurcation. Although it is not unusual to have such high velocities (9.2 m/s) at the entrance of the spiral case, velocities are generally kept below 7 or 8 m/s in penstocks and never get to velocities as high as 13 m/s.

Head losses were estimated to approximately 22.6 m, an increase of 17.0 m compared to one unit at full output. This implies that the maximum output of Units 7 & 8 producing simultaneously would then be limited, due to increased head losses, to approximately 275 MW at maximum reservoir water level and 270 MW at minimum reservoir water level.

Water hammer overpressure is highly dependent on flow velocities. Hence, by doubling the flow rate and the velocities in the existing penstock, it was found that the overpressure more than doubled and that the overall pressure in the penstock would rise above the existing design pressure. For this reason, a bifurcation, upstream of Powerhouse 2, is not technically viable.

4.2.2 Bifurcation located downstream of the existing Unit 7 intake

A bifurcation near the intake would reduce the water hammer overpressure to within the existing penstock design pressure.

However, this solution implies no savings but rather additional penstock costs as a near full length new penstock would have to be provided in addition to the bifurcation. Additional equipment and structures would also be required, including two 5.72 m diameter butterfly isolation valves downstream of the bifurcation, two vents reaching above the maximum reservoir water level downstream of the valves to prevent implosion of the penstock, a concrete block encasing the bifurcation and valves, and a building sheltering the valves.

This alternative does offer the potential saving of a new intake but any cost savings would be negated by the loss of revenues resulting from the required six to eight months shutdown of Unit 7 necessary to build, install and commission the bifurcation and its surrounding equipment.

Moreover, the new power plant (Units 7 & 8) would be left with an existing intake that does not have the required submergence to pass the flow adequately for both units operating at maximum output, thus, limiting the simultaneous use of both units.

In this regard, this alternative is also deemed to be technically unfeasible.

4.2.3 New Unit 8 dedicated penstock alternative

The new Unit 8 dedicated penstock alternative consists of an independent penstock combined with a new intake, connected to the enlarge headrace canal for the sole supply of Unit 8.

All structures for this alternative were designed to allow for the uninterrupted operation of Unit 7 during construction.

The same overall geometry and design of the Unit 7 penstock was considered for this alternative. The head losses and the water hammer were also assumed to be nearly the same as for Unit 7 (head losses at full output of approximately 5.65 m and a water hammer overpressure less than 30% of the gross head).

For this alternative, the rock and land profile were assumed to be generally the same as for the existing Unit 7 penstock, which should not be far from reality since the new penstock will be located 36.5 m aside from existing penstock at the intake and only 18.5 m aside at the powerhouse.

As shown in the Unit 7 as build drawings, the required rock excavations to lay the new penstock near the powerhouse were executed as part of a pre-investment during the construction of Unit 7.

LIDAR as well as soil investigations along the axis of the new penstock should be completed in the next project phase to define the land and rock profile as well as the soil properties.

4.2.4 Recommended alternative

Based on an analysis of the proposed alternatives, a new dedicated penstock with a new intake and enlarged headrace canal is recommended.

The design and profile of the existing Unit 7 penstock was used for the class 3 estimate. The variable plate thicknesses were derived from the existing penstock with the difference that G40.21 350WT steel was used instead of A36, G40.21 300WT and ASTM A 517 grade 700 steel. This allowed for a small reduction of plate thickness at some locations. Finally, the design of the stiffeners along the existing Unit 7 penstock was used for Unit 8 penstock.

Excavation and infill sections quantities used for penstock of Unit 7 were also used for the Unit 8 estimate. A temporary concrete retaining wall is required near the powerhouse so that excavation works for the new Unit 8 penstock do not uncover and destabilize Unit 7 penstock. This wall is included in the estimate.

5 TAILRACE CHANNEL

5.1 Existing tailrace channel with Unit 7 only at full output

Near the powerhouse, the existing tailrace channel is excavated to its final geometry to accommodate 2 units. At the exit of the draft tubes, the channel is 30.5 m wide and excavated in rock with vertical side walls. It narrows from 30.5 m to 15.25 m and its invert rises from elevation -9.5 m to -3.0 m over approximately 40 m while the side walls remain vertical. On the next 70 m, the rock surface drops progressively from elevation 3.0 m to -3.0 m and side slopes vary from vertical to 2H:1V, while the invert remains 15.25 m wide at elevation -3.0 m.

From this point about 110 m downstream of the powerhouse, the channel was narrowed to the width required for one unit only, assuming it would be enlarged at the time of construction of Unit 8. Consequently, the invert width reduces gradually from 15.25 m to 6.1 m over approximately 100 m corresponding to the length of the curve in the canal. For the remaining 150 m to join Powerhouse 1 outlet channel, the canal section does not change with an invert 6.1 m wide at elevation -3.0 m and lateral slopes of 2H:1V.

The water level in the channel is influenced by tidal patterns and varies from a minimum normal elevation of 1.2 m to a maximum elevation of 3.5 m at Powerhouse 1. Head losses in the tailrace channel between Powerhouse 2 and Powerhouse 1 were estimated for one unit at full output (102 m^3/s). It was found that losses varied from 0.05 m at high tide to 0.30 m at low tide for an average head loss of approximately 0.20 m. Velocities ranged from 0.8 m/s at high tide to 2.1 m/s at low tide in the narrow section of the channel.

5.2 Tailrace channel with two units (7 & 8) at full output.

Head losses were calculated for the current tailrace channel with the flow of two units at full output (204 m^3/s). They were found to be approximately 0.25 m at high tide and 1.10 m at low tide for an average head loss of approximately 0.70 m while velocities ranged from 1.6 m/s at high tides to 4.2 m/s at low tides.

If the channel is left as is, there will be an additional head loss of 0.20 m to 0.80 m for an average of approximately 0.50 m when both units are operating at full output simultaneously. This represents an average loss of approximately 0.85 MW. Assuming both units would be operated simultaneously about 3 hours a day and an electricity rate of 0.10 $/kW, this would represent a potential annual revenue loss of approximately $90 000.

To limit head losses and flow velocities during simultaneous operation of both units at maximum output, the narrowest section of the channel would have to be enlarged by 9.15 m to a width of 15.25 m at the actual invert elevation. In that case, head losses would be brought back to an average of 0.2 m and velocities would be limited to 2.1 m/s at low tides. This represents an excavation of approximately 108 000 m^3 of overburden and the placing of 5 000 m^3 of riprap. An order of magnitude estimate for this work is approximately $ 2 000 000.

As there was no riprap placed on the left side of the channel at the time of construction, it is assumed that the original plan was to enlarge the channel and place riprap on the left side of the channel when constructing Unit 8. Placing riprap at that time would have been a waste since it would have to be excavated when enlarging the canal to accommodate Unit 8. Moreover, the size of the actual riprap in place on the right side and the invert of the existing channel was designed for velocities of 2.1 m/s. Considering that, if the canal was not enlarged, velocities would raise up to 4.2 m/s at low tides for two units operating simultaneously at full output, riprap would not only have to be put in place where it is presently absent but existing riprap would also have to be replaced by a new riprap made of larger size rocks to resist erosion at such velocities. This work is estimated to approximately $ 500 000.

The decision as to whether or not to enlarge the channel therefore becomes an economic decision that should be reviewed during a future phase of the project. However, preliminary numbers are indicating that it is worth to invest $1 500 000 more to enlarge the channel and get an annual return of at least $90 000. In this regard it is recommended for the time being to enlarge the tailrace channel.

6 CONCLUSION

6.1 Summary

Following the review of the existing hydraulic conveyance system of Unit 7, it was found that the new Unit 8 will require its own dedicated intake and penstock. It is not feasible to provide water to proposed Unit 8 utilizing the existing intake and penstock of Unit 7 for the following technical reasons:

- Flow velocities in the existing headrace canal would reach up to 0.86 m/s and 1.75 m/s in the upstream and downstream portion of the canal respectively when operating Units 7 and 8 through the same existing intake. At such velocities, it is not possible to maintain a stable ice cover and thus it would add many constraints to the operation during the ice cover period.

- The existing Unit 7 intake was designed for a maximum flow of 102 m^3/s which corresponds to the flow of a single 150 MW unit. Its actual submergence at low reservoir level is already marginal and some intermittent vortex are probably actually present when operating at low water levels. Moreover, at low water levels, floating free pieces of ice and debris are already sucked down on the trash racks, but, because of the submersion at the gate, no air is actually sucked into the penstock. Permanent vortex would be present even at medium water level when using the existing intake for both units and there is a possibility that air could be sucked into the penstock and make its way down to the generating units affecting their efficiency and behaviour. Also, more floating pieces of ice and debris could be sucked down in the trash racks if both units were using the existing intake, creating obstruction and additional head losses at the intake and eventually impeding the operation of the generating units.

- The existing Unit 7 penstock was designed and built to supply one single unit of 150 MW with flow velocities ranging from 4.85 m/s at the intake to 9.2 m/s at the powerhouse for a head loss of 5.65 m and a water hammer overpressure corresponding to 30% of the gross head. If the existing penstock was to be used for both units with a bifurcation near the powerhouse, operating simultaneously both units at maximum output would double velocities, resulting in a head loss of approximately 22.6 m and a water hammer overpressure much higher than the resistance of the actual penstock, thus, technically eliminating this alternative.

Moreover, the implementation of a bifurcation would require a minimum shut down of 6 to 8 months of Unit 7 which would negate any potential cost savings for the bifurcation option.

It is also recommended to enlarge both headrace and tailrace canals in order to improve the efficiency of the water conveyance system when operating both Units 7 and 8.

Although it is technically possible to keep the tailrace canal dimensions as is with the addition of Unit 8, an average additional head loss of 0.5 m would affect the production of both units when operating simultaneously. Velocities would also reach 4.2 m/s at low tides and would require the placement of riprap. In this regard, it is also recommended to proceed with the enlargement of the tailrace canal.

6.2 Recommended works

The following works on the water conveyance system are therefore required to accommodate an additional 150 MW unit:

- Existing headrace canal enlargement:
  - Enlarge the upstream portion of the canal excavated in the overburden by 12.2 m on its north side.
  - Enlarge the downstream portion of the canal excavated in rock by 16.75 m on its south side.
  - Develop a bifurcation in the canal near the intake and excavate in the rock a new approach canal 15.25 m wide for the new intake south of the existing approach channel. Depending on future reservoir and units operating patterns, yet to be determined, some of the excavations at the headrace canal could be reduced or eliminated. Also, A bathymetry and a 2D flow analysis are required upstream of the headrace canal to ascertain the adequacy of flow capacity.
- New Unit 8 Intake:
  - Build a new intake similar to existing Unit 7 intake but with the deck raised by 1.2 m to account for the raised reservoir water levels;
- New Unit 8 Penstock:
  - Build a new penstock similar to Unit 7 penstock from the new Unit 8 intake to the powerhouse. A temporary retaining wall will be required near the powerhouse to prevent unearthing and destabilizing existing Unit 7 penstock during excavation works.
- Existing tailrace channel enlargement:
  - Enlarge the downstream portion of the canal by 9.15 m and place riprap on the left side depending on economic analysis to be done in a future phase. Not enlarging the channel would result in an average additional head loss of 0.5 m and placement of riprap would still be required.

An investigation and survey campaign is required to determine rock and ground profile. Rock coring and soil test will also be required to ascertain the rock quality in the zones of the rock plug and the rock pillar between both entrance channels as well as soil properties in all zones where overburden excavations needs to be performed. The probable cost for those investigations is estimated at 500k$.

Project costs, schedule and impact of the construction of Unit 8 on the operation of Unit 7 are included in the RFP Scope Item 2 report (SLI document 647756-0000-40ER-I-0002-00).